Summary report, Lehigh University, (1954)

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Welded Continuous Frames and Their Components

Progress Report Z

SUMMARY REPORT

Project Staff

(Not for Publication)
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FOREWORD

This summary report is prepared for information and reference at the August 13, 1954 meeting of the Lehigh Project Subcommittee. Some recent results obtained on the project are included, and certain reference material has been added at the end of the report for permanent record.

Projects for which brief reports are not included herein will be presented orally. Two technical reports have recently been distributed:

Progress Report W, Discussion to A.S.C.E. Separate 292 on Elastically Restrained Columns

Interim Report 26, "Rules of Practice in Plastic Design"

This work has been carried out as part of an investigation sponsored jointly by the following: American Institute of Steel Construction; American Iron and Steel Institute; Column Research Council (Advisory); Navy Department-Office of Naval Research (Contract No. 39303), Bureau of Ships, Bureau of Yards and Docks; and the Welding Research Council.

Lynn S. Beedle

Fritz Engineering Laboratory
Lehigh University
Bethlehem, Penna.

12 August 1954

Fritz Laboratory Report No. 205.25
1. OBJECTIVE

The objectives of the project are contained on page 51. Particular emphasis is now being placed on Objective 6, the developing of practical design procedures.

2. PROGRAM OUTLINE

On page 5 are listed the various programs that have been set up as a basis for carrying out the project. For each program the phases involved have been listed, and the arrangement is according to the status of solution (to be described below).

3. STATUS

The evaluation of the results obtained on the various programs has demonstrated the applicability of plastic analysis in structural design. It has therefore been possible to prepare suggested "Rules of Practice" for ordinary structures. (Interim Report 26) Although some of the rules are tentative, they permit significant application of plastic analysis in structural design. Probably the most important single problem that requires attention concerns lateral bracing requirements. Even so, it is considered that plastic design of certain rigid structures such as continuous beams and industrial frames is now appropriate.

The figure on page 6 is taken from a report presented to the National Engineering Conference of the AISC, April, 1954. It is intended to present a picture of the ultimate strength of single span rigid frames in comparison with conventional design. There is plotted to the left the percent of the predicted ultimate strength. Full-scale tests at Lehigh and a group of tests made at the University of Cambridge, England, are shown. The clear portion of each bar graph represents the portion up to the allowable working load. The cross-hatched region represents the loading range between the allowable working load and the computed yield point. Finally, the solid portion represents the observed reserve of strength beyond the computed elastic limit. The dotted line across the chart is a working load based on ultimate strength as the design criterion using a load factor of safety of 1.75.
The applicability of Plastic Design is illustrated by the following observations made in connection with the figure on page 6.

1. The reserve in strength above present conventional working loads is considerable in continuous steel structures (150 to 200%).

2. The reserve of strength above the nominal theoretical yield point is quite large in every case. In some instances as much capacity is disregarded as is used in design.

3. The plastic strength may be predicted quite closely.

4. A possible working load for plastic design provides the same reserve of strength for quite a variety of structures.

5. The increase in working load, made possible by using the plastic design method, is invariably greater than 30%. (In these tests the possible increase ranged from 42% to 90%).

6. Stresses at the working load based on ultimate strength are below the computed elastic limit.

7. Since the allowable load on the simple beam (shown in the figure) is the same for both elastic and plastic design, use of the ultimate strength as the design criterion provides one with at least the same margin of reserve strength as is presently provided in the conventional design of simple beams.

4. FUTURE PLANS

The programs and phases are outlined in the chart on page 5. Those listed in the heading "Current" include those on which work is under way or they are considered important for the coming year. Those phases listed under "Future Planning" are suggested as possibilities, some of which will surely be accomplished; others may be postponed indefinitely. The phases listed under "Work Done" are either partially or entirely completed.
In reviewing plans for the coming year, the following phases seem to be the most important:

A. Practical Applications
   1. Tentative Specifications and/or Rules of Practice
   2. Design Examples
   3. Analysis Procedures

B. Frame Studies
   1. Portal Frame (Gabled roof, combined loading)
   2. Industrial Frame (severe test of plastic behavior)

C. Studies of Components
   1. Columns: Lateral-Torsional Buckling
   2. Corner Connections: Size Effect (complete report)
      Ship-Type Knees
      Haunched Connections or Web Reinforcement
   3. Local Buckling: Limiting $d/w$ web ratios
   4. Lateral Buckling: Critical length ratio
      (Lateral bracing of beams)

Some of these future plans are outlined in specific detail in later parts of this report. General discussion is included in Interim Report 26.

Lynn S. Beedle
**WELDED CONTINUOUS FRAMES AND THEIR COMPONENTS**

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Lehigh Tests

Cambridge Tests

Maximum Observed Load

Yield Load

Conventional Working Load

Simple beam

Ultimate Load

Working Load

(Predicted Ultimate ÷ 1.75 = 57%)

205D T1
205D T2
205D T3
205D T4-a
205D T4-b
FSF 1
FSF 2
FSF 3

40
30
38
36
30
38
36
36
INTRODUCTION

In the determination of the ultimate carrying capacity of any given structural element, it is necessary to first of all evaluate the manner in which the member in question will fail. For the generalized beam-column problem a large variety of such conditions exists, and the following summarizes the various possible modes of failure:

1. Crushing
2. Bending
3. Twist
4. Local Buckling
5. Shear
6. Bending plus Twist
7. Bending plus Local Buckling
8. Bending, Twist and Local Buckling
9. Bending, Twist and Shear

In determining the correct failure type and thereafter evaluating the actual magnitude of the ultimate carrying capacity, certain variables characterizing each individual problem must be considered. The major ones are as follows:

1. Type of Cross-Section
2. Flexural Axis
3. Loading Condition
4. Restraint
5. Slenderness Ratio
6. Size of Member
7. Residual Stresses
8. Relative Amount of Axial Thrust
9. Sequence of Loading

If the member in question is considered as comprising part of a rigid frame, the ultimate strength of which is to be determined, one other variable should be included.

10. Rotation Capacity

Under date of 14 December 1951, a proposal covering column tests was submitted to (and subsequently approved by) the subcommittee. These experiments are now substantially completed, 3 tests remaining in the series (three of the tests were not required). The principal variables included in that program covered

2. Flexure Axis,
3. Loading Condition,
5. Slenderness Ratio, and
7. Residual Stress.
The variables

6. Size of Member,
9. Sequence of Loading, and
10. Rotation Capacity

were also studied in a preliminary way.

From time to time the results of this test program and analytical studies have been presented to the committee (Progress Reports 6, 10, 11 and R).

PRESENT STATUS

The following summarizes the present status of the work on the current Lehigh program:

1. We have been chiefly concerned with the pin-ended column; however, some analytical work has been done on the problem of collapse bending solutions where elastic end restraints are present. (Discussion of ASCE Separate No. 292).

2. Elastic limit solutions to many of the problems have been obtained. (Progress Report No. 6)

3. Collapse solutions have been developed for one loading condition (single curvature) and one type of failure (bending) for both WF and rectangular cross-sections. (Progress Reports No. 10 and 11). These can be extended to other conditions of loading by the methods suggested by Chwalla.

4. It is evident that the case of failure due to bending plus twist is most important. It occurs more frequently than other conditions and for a greater variety of variables. Also, when it occurs there results a lower carrying capacity. (Progress Reports Nos. 6, 10 and R).

5. All as-delivered sections contain residual stresses, and these may have a most profound influence on axial load-carrying capacity as determined by a bending type of failure. (Progress Report No. 10). Preliminary studies (Closing Discussion to Progress Report No. 10) suggest that these stresses also decrease the torsional rigidity of wide flange shapes and thereby reduce carrying capacity as determined by combined bending and twist. Studies are underway to obtain actual collapse solutions for this type of failure.
When the final tests on the approved program have been completed, a technical progress report on the results will be prepared.

**POSSIBLE NEXT STEPS**

Three variables or problems contained in the proposal of 14 December 1951, but for which no tests or analyses were outlined are:

1. Shape (Type of Cross-Section)
2. Sidesway (Loading Condition)
3. Restrained Columns (Restraint)

The problem of failure due to combined bending and twist is introduced by item 1 above.

There seem to be two independent paths which could now be followed. These are best illustrated in Chart I. As will be noted by examination of the Chart both items 3) and 4) could be studied simultaneously; however, as a basis for deciding the next best step, the following comments are made with regard to each main topic or heading in the Chart.

2). **Isolated Pin-Ended Member, Bending Solution**

   a. Even though solutions have been obtained for only one loading condition, these could be extended to other conditions by the methods developed by Chwalla.

3). **Isolated Pin-Ended Member, Bending plus Twist Solution**

   a. Most failures have been of this type (both in isolated columns and in columns of frames).
   b. Failures of this type result in a lower carrying capacity.
   c. Test results are available from the present program.
d. Solution is needed to be able to specify lateral bracing requirements.

e. Some preliminary work has been done. (Progress Report No. 11 and work since then).

4). Plane Frame, Bending Solution

a. Designs can be made which assure that this will be the type of failure. (British Plastic Design Code)

b. Some preliminary work at Lehigh has been done (discussion of ASCE Separate No. 292), and work is actively underway at another institution on the same general problem.

c. Some test results are available from work by other institutions for checking any analytical solution which may be developed.

5). Isolated Pin-Ended Member, Bending Plus Twist Solution Bi-Axial Loading

a. Lehigh Institute of Research has granted an appropriation for fabrication of special end fixtures for model column tests. These will be designed and a pilot series of tests will be carried out as a graduate research project.

7). Space Frame, Combined Bending Plus Twist Solution

Solution to this problem is, of course, one of the ultimate goals of this overall column investigation.

THE PROPOSAL

It is therefore proposed that for the next year the major emphasis be directed toward item 3), the analytical solution of the combined bending plus twist problem for pin-ended members. If the situation permits; work could also start on item 4), collapse bending solutions for plane frames. The problem of sidesway, as such, is here again deferred.

Robert L. Ketter

Lynn S. Beedle
**CHART I**

1). Isolated Pin-Ended Member
   **ELASTIC SOLUTION**

2). Isolated Pin-Ended Member
   **BENDING SOLUTION**
   (Certain specified Loadings)

3). Isolated Pin-Ended Member
   **BENDING PLUS TWIST SOLUTION**

4). Columns in Plane Frames
   **BENDING SOLUTION**

5). Isolated Pin-Ended Member
   **BENDING PLUS TWIST SOLUTION**
   (Bi-Axial Loading)

6). Columns in Plane Frames
   **BENDING PLUS TWIST SOLUTION**

7). Space Frame
   **BENDING PLUS TWIST SOLUTION**
At the conclusion of the first phase of the connection program in which a series of corner connections for 8-inch rolled shapes was tested, a question arose as to whether the same behavior would be observed in larger members. As a result, a series of tests was run to determine the influence of size of member. (Proposal for Phase II dated December 3, 1952.)

Work Completed:

1. Compression tests of type 8B corner connections fabricated from three rolled sections were completed. A fourth member tested earlier was compared with these sections. The sections tested were:

   T - 101  14 WF 30
   T - 102  24 WF 100
   T - 103  36 WF 230
   T - 11   8 B 13 (Earlier test - Connection L)

   The results of these tests are shown on page 14. A sketch showing the layout and welding detail of a type 8B connection is shown on page 15.

2. A number of tension coupons were tested from each member used in the fabrication of specimens. The test results were used in planning the tests of the corner connections.

   Coupon test results will be used to show the variation of properties along flanges, along webs, and between members in the same rolling. On page 17 are tabulated the results obtained to date.

3. Test number T-104 was run on the 14 WF 30 connection in tension. This test is compared with the compression test, T-101, on page 17.

Future Work on the "Size Effect":

1. Reduction of the remaining data and preparation of a progress report on this phase will be completed.

2. A tension test on the 24 WF 100 knee is scheduled in the Fall.

3. Compression coupons will be tested to supplement the available tension coupon data.
Additional connection tests and analyses are outlined elsewhere.

**Preliminary Conclusions:**

1. All the knees tested developed strengths greater than the predicted bending moment.

2. Proper welding procedure and inspection assure development of plastic hinges without fracture of welds in the type of material tested. (No heat treatment was necessary on these connections.)

3. Lateral support to prevent lateral torsional buckling in the plastic range is a very important consideration.

George C. Driscoll, Jr.
First: 3/8 Fillet at root of butt weld.
Second: Chip out and then make butt weld.
45° Fillet 3/16 Legs

- Corner Weld \( \frac{3}{4} \times 1 \)
- Fillet Weld \( \frac{5}{8} \)
- 45° Fillet Weld
- 3/16 Legs

Flange ~ 1 1/2 Thick
Web ~ 3/4 Thick
COMPARISON OF TENSION AND COMPRESSION TESTS
of a
TYPE 8B CONNECTION
14 WF 30
PROJECT 205C

MOMENT AT THE KICK -- PER CENT OF BOILED SECTION YIELD MOMENT

T-104 (Tension)

T-101 (Compression)

Pred. Max. Mom.

Pred. Yield Moment

ATSC Allowable Moment

DEFLECTION -- PER CENT OF PREDICTED YIELD DEFLECTION
### TABLE I

Summary of Tensile Coupon Test Results of Steel Used in Connections

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<td>Upper Yield Point Stress (psi)</td>
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<td>Flange</td>
<td>36,300</td>
<td>36,900</td>
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<td></td>
<td>Web</td>
<td>41,700</td>
<td>42,500</td>
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<td>T-102</td>
<td>Flange</td>
<td>34,900</td>
<td>36,000</td>
</tr>
<tr>
<td></td>
<td>Web</td>
<td>39,800</td>
<td>41,500</td>
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<td>41,300</td>
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<td>T-11</td>
<td>Flange</td>
<td>41,800</td>
<td>47,100</td>
</tr>
<tr>
<td></td>
<td>Web</td>
<td>47,100</td>
<td>47,100</td>
</tr>
</tbody>
</table>
CORNER CONNECTION PROBLEMS

At the last meeting of the Subcommittee a number of the remaining corner connection problems were reviewed. It was agreed that an outline of variables should be submitted for discussion and comment. Based on suggestions received, we would then submit a detailed program for criticism and approval by the committee.

The attached outline of variables (arranged according to "Phases") is therefore submitted to you. Where appropriate, space has been left for your comment at the end of each problem statement. It would be helpful if you would comment on the relative importance of the problem and suggest specific details where possible.

<table>
<thead>
<tr>
<th>Phase I: INFLUENCE OF DESIGN</th>
<th>Types: 2, 2B, 4(a)</th>
<th>Shapes: 8WF31(b)</th>
</tr>
</thead>
<tbody>
<tr>
<td>DETAILS</td>
<td>5A, 7, 8B, 15, 16</td>
<td>14WF30</td>
</tr>
</tbody>
</table>

This phase has been completed and has been reported in Ref. 1. Analysis and tests involving 8B13 rolled shapes were completed.

<table>
<thead>
<tr>
<th>Phase II: SIZE EFFECT</th>
<th>Type: 8B</th>
<th>Shapes: 14WF30</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>24WF100</td>
</tr>
<tr>
<td></td>
<td></td>
<td>36WF230</td>
</tr>
</tbody>
</table>

Tests: Completed as of July, 1954
Analysis: Small amount yet to be done as part of the report.

Preliminary results are presented in the "Summary Report" (P.R.-Z). A final report on results is in preparation (Progress Report Y).

<table>
<thead>
<tr>
<th>Phase III: ROTATION CAPACITY</th>
<th>Type: - -</th>
<th>Shapes: - -</th>
</tr>
</thead>
</table>

Tests: None planned beyond those required for other phases and other programs.
Analysis: To develop methods of predicting the required rotation capacity and to translate into rules for design use.

This study is part of a current dissertation.

<table>
<thead>
<tr>
<th>Phase IV: &quot;TENSION&quot; BEHAVIOR</th>
<th>Type: All</th>
<th>Shapes: All</th>
</tr>
</thead>
</table>

Tension tests of connections which has been previously loaded in compression have been carried out on many of the specimens at Lehigh. A report on these tests was

(a) On one test (P) a 14WF30 was joined to an 8WF31
(b) Connection Types shown on Page

(1) "Connection For Welded Continuous Frames", Progress Report #4 by Toprac, Beedle, and Johnston, Welding Journal, 1951, 1952
presented at the A.W.S. meeting in the fall of 1953. This paper should be published soon in the Welding Journal, and will also include results of "prime" connections tested in tension at the University of Texas. It would help in planning possible tests to know how frequently "positive moment" was encountered at connections in portals.

Comment:

In an actual structure with adjacent columns, tension could occur in the combined column joint under high wind and snow load.

Phase V: WEB REINFORCEMENT Types: Straight, Shape: (Select)

Va: Diagonal Stiffeners

As the stiffness of a knee varies, the distribution of the total rotation of the parts of the frame near the knee varies. The knee might be made so strong that very little rotation can occur within the knee proper thereby causing adjacent beam sections to undergo the entire rotation. In a connection with less web reinforcement, the flange and web are spared severe plastic deformation; but with too little reinforcement the connection will not develop the plastic moment.

The studies carried out under Phase I led to a recommended web diagonal stiffener size. A safe rule has been to use a thickness equal to that of the flange. A series of tests designed to check this recommended size would involve an estimated 4 specimens.

Comment:

It seems to me diagonal thickness equal to flange thickness is too severe, possibly diagonal thickness equal to web thickness sufficient.

Vb: Use of Doublers

This constitutes an alternate to Va, above. Doublers would be used parallel to the plane of the web. (Fig. 9 of Ref. 2) (Type 8C). As few as 2 or 3 tests would be adequate. In planning...

the tests, use would be made of results of Phases I and II.

Comment:

Analytical studies followed by a modest number of tests appear to be adequate to investigate proper proportioning of haunched connections. Although, economically, the tapered haunch seems best suited for frames supporting a single loading condition (conventional design) there may still be a field of application of plastic design in the case of a second loading condition with a limited number of cycles. Appropriate studies are:

(a) Proportions of Type 2B connection to assure formation of hinges adjacent to haunch. (Thickness of inner flange, required depth at connection "throat")

(b) "Wedge-beam" construction -- behavior in frames under "primary" and "secondary" loading.

(c) Proportions of column-haunch assemblies with gradual (curving) transition from column base to girder splice.

(d) Reinforcement of curved knees by use of channels in lieu of inner flange plate.

Concerning Type 4 connections (45° "bracket" type) a few tests have been carried out at Texas(3) in which the amount of stiffening was reduced. Type 14 connections appear suitable and the economic procedure will

(3) "An Investigation of Welded Rigid Connections for Portal Frames" by A. A. Toprac, Welding Journal, January, 1954
probably be to require elastic haunch behavior with hinges forming adjacent thereto.

Comment:

The Type 16 Knee was tested as part of Phase I on the basis of application to ship structure stiffeners. Results were reported in Ref. 1. A different problem that has been mentioned is that of the knee in a main transverse frame bent. Due to the girder depth, this knee must be "built-up". Problems concern web stiffening in the vicinity of lighting or access holes. Analysis of this special application and tests, correlated with Phase VIII, appear to be important.

Comment:

Knees of depth greater than 36" have not yet been studied with regard to plastic behavior. (The study will be correlated with Phase VII) It would help in planning the programs to know a typical depth and flange width vs section depth ratio of such connections.

Comment:
Phase IX: ENCASTLEMENT

Type: --  Shapes: --

Since many steel frames are encased in concrete for various reasons it might be of interest to determine the behavior of the welded knee when encased in concrete. The type of connection used might be one of those tested in previous phases of this project. Any studies would be correlated with lateral bracing studies, since it is expected that one of the principal effects will be to stiffen the connections with respect to lateral buckling.

Comment:

Type 1 connections were tested in a program carried out at the University of Texas. From the results(4) the connections performed in very satisfactory manner and additional tests seem unnecessary at this time.

Comment:

The use of channels placed back to back (see sketch, Type 9) to form a rigid frame knee has been suggested by Amirikian. -- etc. It is believed that the characteristics of this type of connection could be established with a few tests. The double web thickness at the knee appears to afford considerable advantages. Aside from the connection itself it would be of interest to establish the behavior of the channel sections in the plastic region.

Comment:
A separate phase on the influence of shape of cross-section is not included because the associated problems are either a part of the above phases or the "Inelastic Instability" program (205E).

A study of connections using I-shapes has been mentioned previously. Some tests were done at the University of Texas. Because of the relatively greater web thickness, less web stiffening is required. Prior to additional testing it must be demonstrated from the over-all point of view that it would be economical to use I-shapes instead of WF sections. As against the fact that I-shapes are heavier for a given section modulus and that tapered flanges are more difficult to connect there must be weighed the improved performance of knees, the increased resistance to shear and to local buckling, and the increased shape factor.

A separate phase on Lateral Bracing is not included because it is considered as part of a broader study of the lateral support requirements for all parts of rigid frames (205H). However, data is collected on each test, and each study is planned keeping this variable in mind.

Similarly, performance of connections under the Repeated Load or Impact are not included as individual phases but are considered as parts of programs of broader coverage (205G, 205K). In the one case the aim would be to determine how many cycles of load (based on plastic design) a connection will withstand. Solution of the impact problem should find application in structures proportioned to withstand blast load.

Comment:

There may be some advantage in using standard shapes for the narrower flanges and to the place advantages and the range of shape factor as adjusting the flange profile. Developing the comparative advantage would provide useful design for

Two copies of this statement are furnished. As suggested at the beginning, we would like you to return one of them with your comments.

F. W. Schutz, Jr.
Square Knees

Haunched Knees

Curved Knees

PORTAL FRAME KNEES
Summary of the Work Done on Project 205E
(Inelastic Instability)

1. Introduction

In Progress Report T it is shown that the existing theories of buckling of plates in the plastic range do not agree with test results. Furthermore, the predictions of the different theories vary widely, which is basically due to the assumed or derived stress-strain relationships, the derivations being for materials with idealized behavior.

The non-homogeneous yielding process of steel presents another problem. However, in the strain-hardening range the material is again fairly homogeneous. For this case expressions for the buckling strength of steel plates have been derived, based on general stress-strain relationships.

Once a satisfactory solution for the strain-hardening range has been obtained, solutions for the intermediate range can be derived in a way outlined in Progress Report T.

Progress Report X which is now being prepared will present the theoretical derivations together with test results.

2. Theory

When a steel plate is compressed in one direction into the strain-hardening range all the deformation properties may be affected. Hence the tangent-moduli in the direction of the compression and perpendicular to it are possibly different. The same may hold for the Poisson's ratios and also the shearing modulus may be affected.

Take the center plane of the plate as the x-y coordinate plane with the x-axis in the direction of compression, then the relationships between the increments of stress and strain can be written as

\[
\begin{align*}
\frac{\text{d} \varepsilon_x}{\text{d} x} &= \frac{1}{E_{tx}} \frac{\text{d} \sigma_x}{\text{d} x} - \frac{\nu_y}{E_{ty}} \frac{\text{d} \sigma_y}{\text{d} y} \\
\frac{\text{d} \varepsilon_y}{\text{d} y} &= -\frac{\nu_x}{E_{tx}} \frac{\text{d} \sigma_x}{\text{d} x} + \frac{1}{E_{ty}} \frac{\text{d} \sigma_y}{\text{d} y} \\
\frac{\text{d} \gamma_{xy}}{\text{d} x} &= \frac{1}{G_{xy}} \frac{\text{d} \tau_{xy}}{\text{d} x}
\end{align*}
\]

(1)
In Table 1 a summary of the assumed or derived values of $E_{tx}$, $E_{ty}$, $v_x$, $v_y$ and $G_t$ as used in different theories of plate buckling is given.

3. Tests on Wide-Flange Sections

In order to investigate the actual behavior of WF shapes with regard to local buckling, 6 shapes were tested under two extreme conditions:

1. axial compression (Test D1, D2, D3, D4, D5, D6)
2. pure bending (Test B1, B2, B3, B4, B5, B6)

The test set-ups for both kinds of tests are shown in Fig. 1. The length of each specimen was divided into three gage lengths over which the change in length was measured directly with 0.0001" Ames dials. Along the edges of the flanges and the center of the web, deflection measurements were taken as shown in the same figure. For the bending tests the lateral rotation was also measured at the loading points (which were supported against lateral rotation) and near the center line of the beam. The dimensions of all specimens are given in Table II.

Fig. 2 shows the obtained $P/A$ vs $\varepsilon_{av}$ curves from the compression tests and $M/Z$ vs $\varepsilon_{av}$ curves from the bending tests ($\varepsilon_{av} =$ average strain at center of compressed flange)

\[ A = \text{area of cross-section} \]
\[ Z = \text{plastic section modulus} \]

For all tests curves of maximum flange deflection, maximum web deflection and lateral rotation vs average strain were plotted. From these curves the critical strains were obtained. The critical strain is defined as the strain at which the lateral deflection of flange or web starts to increase more rapidly.

The results are summarized in Table III. The critical strains of the flanges are plotted vs the $b/t$ ratio in Fig. 3, showing also the corresponding theoretical curves.

4. Tentative Recommendations for the Geometry of Wide-Flange Shapes

a. Flange Buckling

The requirements for the rotation of a plastic hinge depend on the type of structure and the location of the plastic hinge in the structure.
In Figure 3 all test results are summarized. It is seen that the critical strains increase rapidly near a value of the \( b/t \) ratio of 17. For this value the critical strain of a hinged flange just reaches the strain hardening range.

From a study, which is now being made at Fritz Laboratory on the required rotation capacity of plastic hinges, it is known that in general it is sufficient for the strain of the flanges just to reach the strain hardening range. Furthermore, it is seen from the test results that in this case no rapid drop of the moment occurs.

Therefore it can be recommended tentatively to specify \( b/t \leq 17 \). This will give satisfactory performance of a plastic hinge with respect to local buckling except in special cases of large required rotations producing strains in the flanges far beyond strain hardening.

b. Web Buckling

For the wide flange shapes subjected to pure bending, buckling of the webs did not occur for the range of shapes tested

\[
d'w = 27\text{-}40
\]

\[
d' = d - 2t
\]

However, under pure compression web buckling occurred. Taking as the condition that all of the section can be strained up to strain hardening gives

\[
d'/w \leq 30
\]

besides the above specified value of

\[
b/t \leq 17
\]

<table>
<thead>
<tr>
<th>Tentative Specifications</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compression flanges</td>
</tr>
<tr>
<td>Web subjected to pure bending</td>
</tr>
<tr>
<td>Web subjected to pure compression</td>
</tr>
</tbody>
</table>

* All tested shapes \((d'/w = 27\text{-}40)\) showed satisfactory performance with respect to web buckling and the critical value of \( d'/w \) is expected to be considerably higher than 40.
<table>
<thead>
<tr>
<th>Theory</th>
<th>Et</th>
<th>Et</th>
<th>Et</th>
<th>Et</th>
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<tr>
<td>E</td>
<td>E</td>
<td>E</td>
<td>E</td>
<td>E</td>
</tr>
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<td>(\frac{1}{3}E_{t})</td>
<td>(\frac{1}{3}E_{t})</td>
<td>(\frac{1}{3}E_{t})</td>
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<td>(\frac{E}{2(1+v)})</td>
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<td>(\frac{Et}{2E})</td>
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</tr>
<tr>
<td>(\frac{1}{2} + \frac{3}{2}E_{sec})</td>
<td>(\frac{1}{2} + \frac{3}{2}E_{sec})</td>
<td>(\frac{1}{2} + \frac{3}{2}E_{sec})</td>
<td>(\frac{1}{2} + \frac{3}{2}E_{sec})</td>
<td>(\frac{1}{2} + \frac{3}{2}E_{sec})</td>
</tr>
<tr>
<td>(\frac{E}{E_{t}(2v-1)+E})</td>
<td>(\frac{E}{E_{t}(2v-1)+E})</td>
<td>(\frac{E}{E_{t}(2v-1)+E})</td>
<td>(\frac{E}{E_{t}(2v-1)+E})</td>
<td>(\frac{E}{E_{t}(2v-1)+E})</td>
</tr>
</tbody>
</table>
**TABLE II**

Dimensions of Specimens

<table>
<thead>
<tr>
<th>Spec.</th>
<th>Shape</th>
<th>$A^2$ in$^2$</th>
<th>$Z$ in$^3$</th>
<th>$b$ in</th>
<th>$t$ in</th>
<th>$d$ in</th>
<th>$w$ in</th>
<th>$L$ in</th>
<th>$L_1$ in</th>
<th>$C$ in</th>
<th>$b/t$</th>
<th>$d'/w$</th>
<th>$d/w$</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1 D1</td>
<td>10 WF 33</td>
<td>9.66</td>
<td>38.56</td>
<td>7.95</td>
<td>0.429</td>
<td>9.80</td>
<td>0.294</td>
<td>32</td>
<td>32</td>
<td>38</td>
<td>18.5</td>
<td>30.4</td>
<td>33.3</td>
</tr>
<tr>
<td>B2 D2</td>
<td>8 WF 24</td>
<td>6.83</td>
<td>22.56</td>
<td>6.55</td>
<td>0.383</td>
<td>8.01</td>
<td>0.236</td>
<td>26</td>
<td>26</td>
<td>38</td>
<td>17.1</td>
<td>30.7</td>
<td>33.9</td>
</tr>
<tr>
<td>B3 D3</td>
<td>10 WF 39</td>
<td>11.34</td>
<td>45.63</td>
<td>8.02</td>
<td>0.512</td>
<td>9.88</td>
<td>0.328</td>
<td>32</td>
<td>32</td>
<td>48</td>
<td>15.6</td>
<td>27.0</td>
<td>30.1</td>
</tr>
<tr>
<td>B4 D4</td>
<td>12 WF 50</td>
<td>14.25</td>
<td>70.28</td>
<td>8.18</td>
<td>0.620</td>
<td>12.19</td>
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<td>32</td>
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<td>13.2</td>
<td>31.2</td>
<td>34.7</td>
</tr>
<tr>
<td>B5 D5</td>
<td>8 WF 35</td>
<td>10.00</td>
<td>33.68</td>
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<td>0.476</td>
<td>8.13</td>
<td>0.308</td>
<td>32</td>
<td>32</td>
<td>44</td>
<td>17.0</td>
<td>23.3</td>
<td>26.4</td>
</tr>
<tr>
<td>B6 D6</td>
<td>10 WF 21</td>
<td>5.84</td>
<td>22.45</td>
<td>5.77</td>
<td>0.318</td>
<td>9.82</td>
<td>0.232</td>
<td>26</td>
<td>26</td>
<td>30</td>
<td>18.2</td>
<td>39.6</td>
<td>42.3</td>
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</table>

$d' = d - 2t$
### TABLE III

**Test Results**

<table>
<thead>
<tr>
<th>Test</th>
<th>$\sigma_Y$ ksi</th>
<th>$\epsilon_{cr} \times 10^3$</th>
<th>Flange</th>
<th>Web</th>
<th>$\sigma_{cr}$ ksi</th>
</tr>
</thead>
<tbody>
<tr>
<td>D 1</td>
<td>34.4</td>
<td>8.5</td>
<td>8.5</td>
<td></td>
<td>34.2</td>
</tr>
<tr>
<td>D 2</td>
<td>34.0</td>
<td>13.5</td>
<td>13.0</td>
<td></td>
<td>34.0</td>
</tr>
<tr>
<td>D 3</td>
<td>35.2</td>
<td>17.5</td>
<td>17.5</td>
<td></td>
<td>38.5</td>
</tr>
<tr>
<td>D 4</td>
<td>35.0</td>
<td>18.5</td>
<td>6.0</td>
<td></td>
<td>36.8</td>
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<tr>
<td>D 5</td>
<td>36.6</td>
<td>15.0</td>
<td>15.0</td>
<td></td>
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<td>2.3</td>
<td>1.6</td>
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<td>B 6</td>
<td>14.0</td>
<td></td>
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<td></td>
<td>14.0</td>
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</tbody>
</table>
Fig. 1. Test Set-ups
1. Introduction

During the last two years a considerable number of tests have been carried out to investigate the local buckling problem.

First short rectangular columns were tested which buckled in the strain-hardening range. It was shown that Shanley's tangent-modulus theory was applicable to the strain-hardening range of steel (Progress Report S) (1)*

The tests on angle specimens, which buckled torsionally, showed that none of the available theories of plate buckling in the plastic range was applicable (Progress Report T) (2).

Finally tests on WF-sections under pure compression and pure bending have been carried out. The results will be reported in Progress Report X which is now being prepared. A summary of the results will be presented at the next meeting of the Lehigh Project Subcommittee. Furthermore, an attempt has been made to formulate the local buckling problem in terms of general stress-strain relationships.

The following instability problems remain to be investigated:

a. Buckling of webs of WF-sections under pure moment

Tests have been carried out on beams with depth over thickness ratios of the webs up to 40. No web buckling occurred and it is expected that the limiting ratio will be considerably higher than 40.

* See list of references at the end of the report.
b. **Buckling of flanges and webs under moment gradient**

Up till now all bending tests were on WF-sections under pure moment. For the flanges this is probably the most severe loading condition and tests may reveal that under a moment gradient a higher width over thickness ratio could be allowed. In the case of web buckling this loading condition may be more severe than pure bending due to the influence of shear.

c. **Stiffening of WF-sections**

If the tests reveal that a large number of available shapes would perform unsatisfactory, the effect of stiffening devices should be investigated.

d. **Lateral buckling of WF-beams**

The bending tests which were carried out during the last year revealed that failure mostly occurred due to a combination of local and lateral buckling. In order to be able to specify the spacing of lateral supports more information is needed.

To investigate these remaining problems more tests are required than possibly could be carried out in one year. Therefore, it is proposed to investigate problems a and d during the coming year. The information which will be obtained from these tests is most urgently needed at the moment.

2. **Buckling of Webs Under Pure Moment**

The purpose of these tests is to determine the limiting depth over thickness (d/w) ratio for which a WF beam can be bent such that the strain of the flanges reaches strain-hardening without occurrence of web buckling.
Fig. 34 of Interim Report No. 26 (3) shows the cross-section dimensions of WF, Light and Junior beams. From this figure it is seen that if the limit is as high as \( d/w = 50 \) most of the available shapes would be within this limit. Therefore, first a bending test on a 12 WF 27 with \( b/t = 16.25 \) and \( d/w = 49.79 \) will be carried out. Depending on whether or not the performance is satisfactory another section with \( d/w > 50 \) or one with \( d/w < 50 \) will be tested. Finally a section with \( d/w > \) limit and \( b/t = 1.4 \) will be tested in order to investigate the stiffening effect of the flanges on the web.

Thus a total of about 3 tests will be carried out. The test set-up will be the same as for the bending tests performed during the past year.

3. **Lateral Buckling of WF Beams Under Constant Bending Moment**

   a. **Theory**

Suppose a WF beam is bent such that the strains of the flanges have reached the strain-hardening range. Then the relationships between increments of stress and strain for the material of the flanges are

\[
\frac{\partial \varepsilon_x}{\partial \sigma_x} = \frac{1}{E_t} \quad , \quad (E_t = \text{tangent modulus})
\]

\[
\frac{\partial \gamma_{xy}}{\partial \tau_{xy}} = \frac{1}{G_t} \quad (G_t = \text{shear modulus in the strain-hardening range})
\]

(x-axis is taken in the direction of the axis of the beam)

For the beginning of strain-hardening

\[ E_t = E_{st} = 900 \text{ ksi} \]

\[ G_t = G_{st} = 2,500 \text{ ksi} \]
The value for $G_{st}$ has been obtained from the results of angle tests as reported in Progress Report T (2).

Under the assumption that lateral buckling occurs under an increase in moment an expression for the critical moment can be derived in the same way as in case of elastic lateral buckling. Derivations for this elastic case can be found in the pertinent literature e.g. Timoshenko (4).

For the beginning of strain-hardening the critical moment is

$$M_{cr} = \frac{\pi \sqrt{E_{st} I_y G_{st} K}}{k l} \sqrt{1 + \frac{\pi^2 a^2}{k^2 l^2}} \quad (2)$$

with

- $I_y$ = moment of inertia about weak axis
- $K$ = torsional constant
- $a^2 = \frac{E_{st} I_y h^2}{4 G_{st} K}$
- $h = d - t$
- $d$ = depth of section
- $t$ = thickness of flange.

The factor $k$ is a constant depending on the end conditions.

b. Test set-up

A test set-up has been selected with boundary conditions which can be easily realized. The set-up is shown in Fig. 1. A beam is loaded with two concentrated loads. The center part, being subjected to a constant bending moment, is the actual test specimen. The ends of the beam are made into a box-section by welding plates against the WF shape. This way a high torsional rigidity and moment of inertia about the Y-Y
axis are obtained. Furthermore the beam is supported against lateral rotation at the loading points.

Calling $\beta$ the angle of twist, then the boundary conditions at the loading points for the above described test set-up are

$$\beta = 0 \quad \text{and} \quad \frac{d\beta}{dx} = 0.$$  

c. Proposed Tests.

For the first lateral buckling tests section 10MF29 has been selected with

- $b/t = 11.6$
- $d/w = 35.3$

These values of the $b/t$ and $d/w$ ratios ensure that no local buckling of flange or web will occur.

From eq.2 the critical length can be computed for which the beam will buckle laterally just when the strains of flanges reach strain-hardening. With $k = 2$ for this proposed test set-up the critical length for the 10MF29 becomes

$$L_{cr} = 2 \frac{L_{cr}}{L_{cr}} = 50\text{"}.$$  

Three tests on this section are proposed

1. $L = 48\text{"}$
2. $L = 72\text{"}$
3. $L = 96\text{"}$

However the length of test specimens 2 and 3 may be altered according to the results from test 1.

Finally a section with limiting values of the $b/t$ and $d/w$ ratios and $L = L_{cr}$ will be tested. This section will be selected according to the findings of the previous tests.
Thus a total of about 4 tests will be carried out.

4. Summary of Proposed Tests

Web buckling under constant moment

<table>
<thead>
<tr>
<th>Test</th>
<th>Shape</th>
<th>b/t</th>
<th>d/w</th>
</tr>
</thead>
<tbody>
<tr>
<td>W 1</td>
<td>12WF27</td>
<td>16.25</td>
<td>49.79</td>
</tr>
<tr>
<td>W 2</td>
<td>?</td>
<td>16-17</td>
<td>&gt;50 or &lt;50</td>
</tr>
<tr>
<td>W 3</td>
<td>?</td>
<td>14</td>
<td>&gt;limit</td>
</tr>
</tbody>
</table>

The objective of tests W 1 and W 2 is to determine the limiting value of d/w. Test W 3 will be carried out in order to investigate the stiffening effect of the flange on the web.

Lateral buckling under constant moment

<table>
<thead>
<tr>
<th>Test</th>
<th>Shape</th>
<th>L</th>
<th>b/t</th>
<th>d/w</th>
</tr>
</thead>
<tbody>
<tr>
<td>L 1</td>
<td>10WF29</td>
<td>L_{cr} = 48&quot;</td>
<td>11.6</td>
<td>35.3</td>
</tr>
<tr>
<td>L 2</td>
<td>10WF29</td>
<td>72&quot;</td>
<td>11.6</td>
<td>35.3</td>
</tr>
<tr>
<td>L 3</td>
<td>10WF29</td>
<td>96&quot;</td>
<td>11.6</td>
<td>35.3</td>
</tr>
<tr>
<td>L 4</td>
<td>?</td>
<td>L_{cr}</td>
<td>17</td>
<td>limit</td>
</tr>
</tbody>
</table>

Tests L1, L2 and L3 are designed to determine:
1st the critical length L_{cr}
2nd the reduction of the critical strain of the flange when L > L_{cr}.

Test L4 is designed in order to check all previous findings for a section with limiting values for L, b/t and d/w.
List of References

(1) Haaijer, G. "Compression Tests on Short Steel Columns of Rectangular Cross-Section" Progress Report S, Fritz Laboratory, Lehigh University, June 15, 1953.

(2) Thurlimann, B. "Buckling of Steel Angles in the Plastic Range" Progress Report T Fritz Laboratory, Lehigh University, August 15, 1953.


SPECIAL STUDIES

The following special studies are being carried out for course credit by graduate students at the Fritz Laboratory:

Influence of Shear on the Plastic Moment (205B)

To correlate with analysis, a series of tests is to be performed on WF beams on the influence of shear stresses on the plastic moment. Previous tests tend to confirm the theoretical findings, but a program controlled to study the particular variable is desirable.

Preliminary Study of Deflection Stability (205G)

Little is known about the plastic behavior of structures under variable loading conditions. Certain theoretical considerations indicate that progressive collapse by increasing deflection may occur under a limited number of load applications. Tests on WF beams are nearing completion.

Biaxially-Loaded Steel Columns (205A)

A set of end fixtures for testing model columns (3/4-in square and under) will be fabricated to permit an exploratory program of tests to be carried out on concentrically or eccentrically loaded columns. The end conditions will be controlled so that bending about either or both principal axes will be possible.

Bolted Connections in Structures Proportioned by Plastic Methods

In order to explore the possibilities of shop welding and field bolting for appreciable moment resistance, a theoretical study will be made and several alternate designs of bolted connections will be tested to determine dependable "hinge moments".

Shearing Modulus in the Plastic Range (241)

Theories of local buckling depend to a large extent on the plastic shearing modulus. Tests to obtain this data are being conducted on steel tubes under combined compression and twist.

Aging and the Strength of Steel Beams (238)

Tests are being completed in which the specimens have been loaded into the plastic region, unloaded, and after aging have been deformed to maximum load. Different strain rates and aging times have been used to study the influence of these variables on the M-Ø relationship.
WELDED CONTINUOUS FRAMES AND THEIR COMPONENTS

LIST OF REPORTS

I. Progress Reports, Published or for Publication

1. Luxion and Johnston, B. G.
   Progress Report No. 1
   PLASTIC BEHAVIOR OF WIDE FLANGE BEAMS
   Welding Journal, November 1948, p. 538s.

2. Beedle, Ready and Johnston, B. G.
   Progress Report No. 2
   TESTS OF COLUMNS UNDER COMBINED THRUST AND MOMENT

3. Yang, Beedle, and Johnston, B. G.
   Progress Report No. 3
   PLASTIC DESIGN AND THE DEFORMATION OF STRUCTURES
   Welding Journal, July 1951, p. 348s.

4. Topractsoyiou, Beedle, and Johnston, B. G.
   Progress Report No. 4
   CONNECTIONS FOR WELDED CONTINUOUS PORTAL FRAMES
   Part I - Test Results and Requirements for Connections
   Welding Journal, July 1951, p. 359s.
   Part II - Theoretical Analysis of Straight Knees
   Welding Journal, August 1951, p. 397s.
   Part III - Discussion of Test Results and Conclusions
   Welding Journal, November 1952, p. 543s.

5. Yang, Beedle, and Johnston, B. G.
   Progress Report No. 5
   RESIDUAL STRESS AND THE YIELD STRENGTH OF STEEL BEAMS
   Welding Journal, April 1952, p. 205s.

6. Ketter, Beedle, and Johnston, B. G.
   Progress Report No. 6
   COLUMN STRENGTH UNDER COMBINED BENDING AND THRUST
   Welding Journal, December 1952

7. Ruzek, Knudsen, Johnston, E. R., and Beedle
   Progress Report No. 7
   WELDED PORTAL FRAMES TESTED TO COLLAPSE
   Mimeographed February 20, 1952
   Publication scheduled in Proceedings SESA

8. Johnston, B. G., Yang and Beedle
   Progress Report No. 8
   AN EVALUATION OF PLASTIC ANALYSIS AS APPLIED TO STRUCTURAL
   DESIGN
   18 September 1952
9. Yang, Knudsen, Johnston, B. G., and Beedle
Progress Report No. 9
PLASTIC STRENGTH AND DEFLECTIONS OF CONTINUOUS BEAM

10. Ketter, Kaminsky, and Beedle
Progress Report No. 10
PLASTIC DEFORMATION OF WF BEAM COLUMNS
To be published as ASCE-Separate

11. Ketter, Robert L.
A VIRTUAL DISPLACEMENT METHOD FOR DETERMINING THE
STABILITY OF BEAM COLUMNS ABOVE THE ELASTIC LIMIT
Submitted for publication as an ASCE Separate.

- Beedle
RESEARCH ON RIGID FRAMES

- Beedle, L. S.
RECENT TESTS OF RIGID FRAMES
Presented at AISC National Engineering Conference
4/13/54, Milwaukee, Wisconsin.
II. Progress Reports Not for Publication

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   PLANS FOR CONNECTION AND COLUMN TESTS
   November 26, 1948

B. Yang
   PLASTIC BEHAVIOR OF CONTINUOUS BEAMS
   May 26, 1949

C. Chen
   STRENGTH OF COLUMNS UNDER COMBINED BENDING AND COMPRESSION
   May 27, 1949

D. Ruzek and Topractsoglou
   TEST OF A RIGID FRAME KNEE
   June 1, 1949

E. Topractsoglou, Ruzek, and Beedle
   WORKING DRAWINGS FOR THREE CONNECTION TESTS. PROPOSAL FOR ADDITIONAL TESTS
   June 1, 1949

F. Beedle
   GENERAL SUMMARY REPORT
   July 19, 1949

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   STRUCTURAL RESEARCH AT CAMBRIDGE UNIVERSITY
   January 30, 1950

H. Beedle and Yang
   DISCUSSION OF "FLEXURE OF I-SECTIONS ABOVE PLASTIC RANGE"
   By W. H. Weiskopf
   February 20, 1950

I. 205 Staff
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   March 8, 1950

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   SOME RESULTS OF COLUMN TESTS. PROPOSED PROGRAM
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   INTERACTION CURVES FOR COLUMNS
   April 20, 1951
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M. 205 Staff
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CONTRIBUTION TO THE PROBLEM OF ULTIMATE CARRYING CAPACITY
OF SIMPLE AND CONTINUOUS BEAMS OF STRUCTURAL STEEL AND
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October 25, 1951

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September, 1952

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INELASTIC LOCAL BUCKLING OF WIDE-FLANGE SECTIONS
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R. Ketter and Beedle
MOMENT ROTATION CHARACTERISTICS OF BEAM-COLUMNS
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S. Haaijer
COMPRESSION TESTS ON SHORT STEEL COLUMNS OF RECTANGULAR
CROSS-SECTION
June, 1953

T. Thurlimann and Haaijer
BUCKLING OF STEEL ANGLES IN THE PLASTIC RANGE
August, 1953

U. Schutz, Schilling, and Beedle
COLLAPSE STRENGTH OF A WELDED SINGLE BAY FRAME
August, 1953

V. Ketter
A VIRTUAL DISPLACEMENT METHOD FOR DETERMINING THE
STABILITY OF BEAM COLUMNS ABOVE THE ELASTIC LIMIT
F. L. Report #205A.14, March, 1954

W. Ketter and Beedle
Discussion of A.S.C.E. Separate #292, "STRENGTH OF
COLUMNS ELASTICALLY RESTRAINED AND ECCENTRICALLY LOADED"
Submitted for publication to A.S.C.E. - July, 1954

X. Haaijer and Thurlimann
LOCAL BUCKLING OF WIDE-FLANGE SHAPES

Y. Driscoll
CONNECTION BEHAVIOR AS INFLUENCED BY SIZE OF MEMBER
In Preparation
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SUMMARY REPORT
August, 1954

INTERIM REPORTS

26. Beedle and Johnston
RULES OF PRACTICE IN PLASTIC DESIGN
August, 1954
NOMENCLATURE and TERMINOLOGY

A = Area of Cross-section
B = \sigma-axis intercept of extrapolated strain-hardening modulus
b = Flange width
d = Depth of section
E = Young's modulus of elasticity
Es = Secant modulus
Est = Strain-hardening modulus = \frac{\sigma_0}{\epsilon_0}
Et = Tangent modulus
e = Eccentricity
F = Load Factor of Safety
f = Shape factor = \frac{M_p}{M_y} = \frac{Z}{S}
G = Modulus of elasticity in shear
I = Moment of inertia
I_e = Moment of inertia of elastic part of cross-section
KL = Effective (pin-end) length of column. K = Euler length factor
k = \sqrt{P/EI}
L = Span length. Actual column length
M = Moment
M_0 = End moment; a useful maximum moment
M_p = Full plastic moment
M_{pc} = Plastic hinge moment modified to include the effect of axial compression
M_s = Maximum moment of a simply-supported beam
M_y = Moment at which yield point is reached in flexure
M_{yc} = Moment at which initial outer fibre yield occurs when axial thrust is present
P = Concentrated Load
p = Distributed load per unit of length

P_cr = Useful column load. A load used as the "maximum column load".

P_e = Euler buckling load. \( P_e = \pi^2 \frac{EI}{L^2} \)

P_f = Full load

P_p = Full plastic load on a structure computed by simple plastic theory.

P_r = Reduced modulus load

P_s = Stabilizing ("shakedown") load

P_t = Tangent modulus load ... the load at which bending of a perfectly straight column may commence. \( P_t = \frac{\pi^2 E_t I}{L^2} \)

P_y = Axial load corresponding to yield stress level; \( P = A\sigma_y \)

R = Rotation capacity

r = Radius of gyration

S = Section modulus, \( \frac{I}{c} \)

t = Flange thickness

V = Shear force

u, w, \( \delta \) = Displacements in x, y, and z directions

\( M \) = Total distributed Load

w = Web thickness

x = Longitudinal coordinate

y = Transverse coordinate

Z = Plastic modulus, \( Z = \frac{M_p}{\sigma_y} \)

Z_e = Plastic modulus of elastic portion

Z_p = Plastic modulus of plastic portion

z = Lateral coordinate

\( \delta \) = Deflection

\( \epsilon \) = Strain
\( \varepsilon_{st} \) = Strain at strain-hardening
\( \theta \) = Measured angle change; rotation
\( \mu \) = Poisson's ratio
\( \sigma \) = Normal stress
\( \sigma_{ly} \) = Lower yield point
\( \sigma_p \) = Prop. limit
\( \sigma_{uy} \) = Upper yield point
\( \sigma_y \) = Yield stress level
\( \tau \) = Shear stress
\( \phi \) = Rotation per unit length, or average unit rotation; curvature

**Bifurcation of Equilibrium Position**

The phenomenon by which there are neighboring positions of equilibrium to the straight configuration.

**Buckling Load**

The point at which bifurcation becomes possible. It has theoretical meaning; it is not defined experimentally except that it suggests the bent configuration.

**Critical Load**

Limit of structural usefulness. The maximum load a column will carry without too much deflection.

**Maximum Load**

The maximum load a column will carry.

**Effective Length of a Column**

The length of the member between points of inflection.

= KL
PROJECT OBJECTIVES

The original objectives, approved at the March, 1950, meeting of the Lehigh Project Subcommittee are as follows:

1. To determine the behavior of steel beams, columns and continuous welded connections with emphasis on plastic behavior, and to develop theories to predict such behavior.

2. To determine how to proportion various types of welded continuous frames to develop the most balanced resistance in the plastic range so that the greatest possible collapse load will be reached.

3. To determine procedures of analysis that will enable one to calculate the collapse loads of welded continuous frames and to verify the analysis by suitable tests.

4. To determine procedures of analysis that will enable one to calculate the elastic and permanent deformations in welded continuous frames in the range intermediate between elastic limit and collapse load.

5. To explore limitations in the application of plastic range design over and above deformation limitations, namely, fatigue, local buckling, lateral buckling, etc.

6. To develop practical design procedures for the utilization of reserve plastic strength in the design of continuous welded frames.

In brief, then, the program consists of:

1. Column, Beam, and Connection Studies (Frame Components)

2. Frame Studies (Integral Behavior)

3. Practical Applications (Methods of analysis and design with due regard to limitations such as fatigue, deflections, local buckling, etc.)